STRUCTURAL ASSESSMENT OF THE 16th CENTURY COASTAL WATCHTOWERS IN THE DEFENSE SYSTEM OF THE PONTIFICAL STATE

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Abstract: Since medieval times the Mediterranean area and in particular its coasts witnessed a series of clashes for the economic and cultural hegemony, culminated, but not definitely concluded, with the Battle of Lepanto in 1571. The deriving state of incertitude and endless terror among the population pushed military engineers to develop organized coastal defense systems consisting of a network of strongholds such as watchtowers, castles and fortresses whose structural and functional characteristics changed sensitively during the centuries. The result is a vast, heterogeneous and almost unexplored part of our built heritage. This paper focuses on the description of the coastal defensive system of the Pontifical State, concentrating on the geometrical, material and constructive characteristics of a particular structural typology: the 16th century watchtower. The aim is to establish an idealized model validated by historical information and in-situ surveys. A structural analysis is carried out, including a simple index analysis, linear dynamic and nonlinear static analyses. These analyses provide preliminary insights on the global mechanisms of failure and on the real behavior of the structure. Conclusions are drawn concerning the necessity of interventions in the perspective of an appropriate reuse of the towers. which could endorse the restoration and maintenance bringing them to a new life.

Keywords: **historical construction/ masonry/ conservation/ structural analysis/ damage/**

1. Introduction

Nowadays over two thirds of the Italian population inhabits the coastline. For centuries, however, that part of land was left unmanned and deserted, stared with preoccupation and mistrust. From the sea, together with exotic goods and foreign visitors, devastating raids and terrible events would often come. In this context, at the apex of Turkish rule in the Mediterranean region (in the 1500s), the Pope's coastal defense saw the proliferation of the 16th century watchtower (referred to as *Torre Pontificia [1]*). Such proliferation embodied the tangible proof of how acute the issue of invasions had become and how concerned but resolute the local administrators were in trying to tackle it. This research attempts to fill a knowledge gap concerning the structural performance of the coastal defense system of the Pontifical States in the perspective of a more feasible restoration of the many public and private properties which still await for rehabilitation. More specifically the study focuses on the 16th century towers aiming at establishing an idealized model which could provide insights on their structural behavior. These structures are the final result of a long optimization process of military architecture to the growing use of firearms during that period. Differently from other constructions belonging to the system, the 16th century watchtowers constitute the very first case of application of a standard design (i.e. same geometry, materials specifications and constructive details, see section 3) and can be classified under a unique, formal typology. More than fifteen towers which still exist today belong to such type. This postulates the possibility to scale up the results obtained from a structural analysis of a purposely prepared model to a number of real structures, determining hence their global safety and consequently concluding about their reuse and strengthening necessities [2].

The paper can be subdivided into the following parts: section 2 is dedicated to introducing the overall defensive network of the Pontifical States, with particular attention to its historical and geographical development; section 3 concerns the proposal of an idealized model representative of the material and geometrical characteristics of the 16th century watchtowers; section 4 concentrates on the study of the structural behavior of the tower based on the assessment of simple geometrical indexes and on linear dynamic and non-linear static analyses, indicating the earthquake resistance and the main vulnerabilities of the construction.

2. A glimpse to the coast: analysis of the defense system

The defensive system has its focus in the city of Rome and it is fully included within the borders of the Latium region, central Italy. The system counts almost sixty structures organized in a chain-like pattern along the Tyrrhenian Sea coast, for a total of 250km (fig.1). As a general rule, the structures present a maximum distance of 10-20km, considered as optimal spacing for prompt communication. Such rule ascertains that some structures served solely, or primarily, as communication bridges shortening the distances and accelerating the passing on of information rather than as centers of active defense. Although the majority of constructions is located directly on the coast, older structures are also found on in-land hills (for alarming the population in case of incursions) and on rivers or secondary water streams leading deep inside the Pontifical territories (as on the Tiber river).

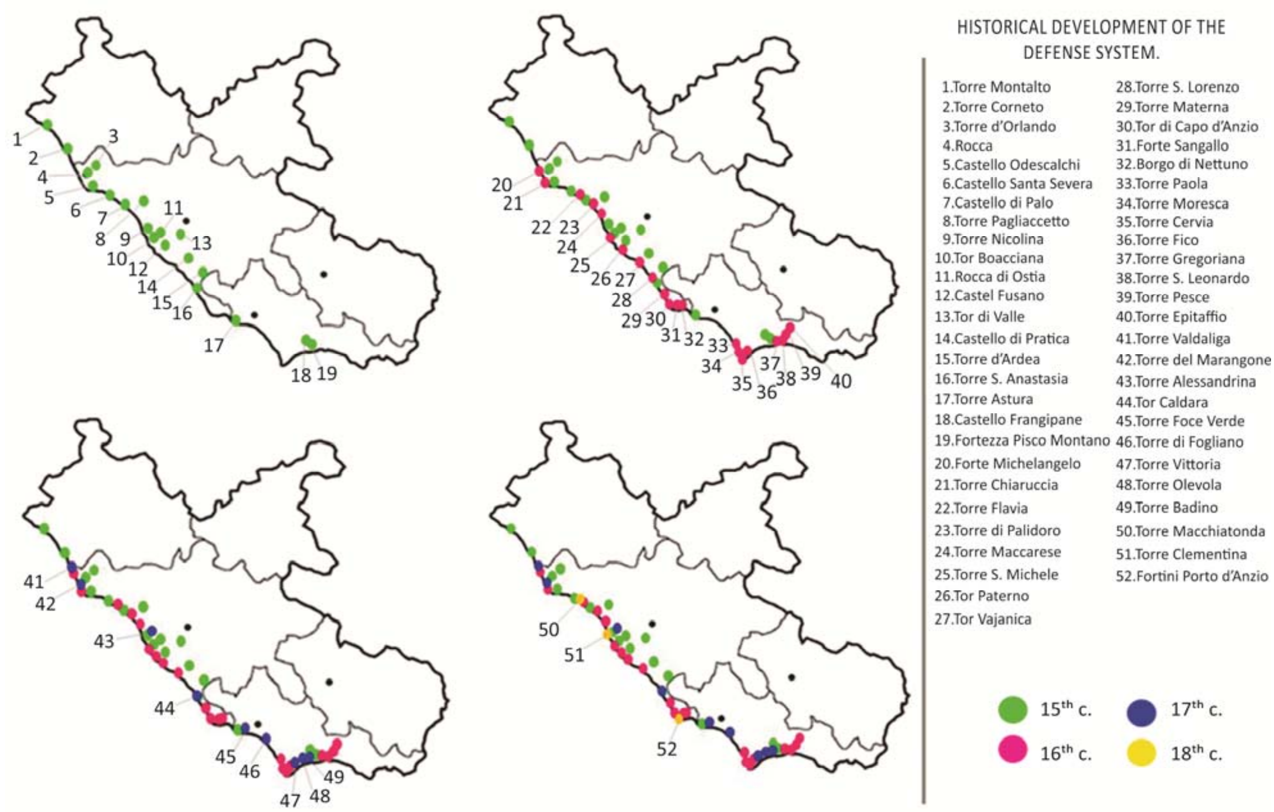

Figure 1. Development of Defense System in Latium.

The building typologies included in the network vary from fortresses to castles and watchtowers. Fortresses usually present the main great-tower, either of cylindrical or square plan, and the perimeter bastions. These are located mainly at strategic spots such as ports (Michelangelo's Fort (1535) in Civitavecchia; Sangallo Fort (1501-1503) in Nettuno) or at outlet of important river (Julius II Fort (1483-1487) in Ostia, on the Tiber). The castles belonging to the defense system are a structural development of medieval towers (11th-12th century), fortified and strengthened during the centuries. Examples include Odescalchi Castle in Santa Marinella and the Santa Severa Castle, established on a 10th century tower with the placement of a cylindrical tower ('Saracen Tower') in the 16th century. Watchtowers constitute the most widespread building typology along the coast. These consist in free-standing structures whose geometry varies depending on the period of construction and location.

The evolution of the defensive system offers an interesting perspective on the economic and political development of the Pontifical States as the alternation of periods of proliferation of watchtowers with moments of abandonment, which coincide symptomatically with the ascent and decline of the Papacy. Although some attempts to introduce a primordial defense system took place during the 12^{th}- 13^{th} centuries, it is only with the Turks hegemony in the 1500s that the constitution of a uniform coastal defense system of the Pontifical State became impellent. With the *'Consitutio de aedificandis turribus in oris maritimis' (1567)*, by Pius V (1566-1572), the effort to institute a homogeneous and organized defensive network was formalized. In this period sixteen new towers were built. Four towers were inserted in the stretch of coast between the well-fortified Civitavecchia and Ostia. *Torre Chiaruccia* (1570-80) was built to connect the port of Civitavecchia to the castles Odescalchi and Santa Severa; *Torre Flavia* (1568) put in communication the Santa Severa castle with the Palo castle and finally *Torre Palidoro* (1562) and *Torre Maccarese* (before 1567) connected Palo to the outlet of the Tiber and therefore to the city of Rome. Other towers were built to substitute those that became inefficient, either due to their state of decay or due to the withdrawn of the sea or the deviation of river waters, as in the case of *Torre San Michele* designed by Michelangelo in 1560 which absorbed the custom function of *Tor Boacciana* (medieval tower restored in 1420). The North part of the defensive system was also connected to the important center of Anzio through *Torre di San Lorenzo* (1570), *Tor Materna* (dating not known, probably between 16th and 17th century) and *Torre di Capo d'Anzio* (medieval tower restored in 1560). Two other stretches of the coast were strengthened: on the wild Circeo promontory (*Torre Paola* 1563, *Torre Moresca* 1562, *Torre Cervia* 1563 and *Torre Fico* 1563) and the gulf of Terracina (*Torre Gregoriana* 1584, *Torre S. Leonardo* 1590 ca., *Torre Pesce* between 1550-1600 and *Torre Epitaffio 1570*) out-looking south and communicating with the defense system of the Kingdom of Naples. After the defeat of the Ottoman forces (1571) the commitment to create an integrated defense system halted and slowly vanished. However, in the 17^{th} century the net of fortifications was tied up by new constructions to shorten the average distance between the towers. *Torre Valdaliga* (1616) and *Torre Marangone* (1605) were introduced to better communicate between the farthest North towers (Montalto and Corneto) with Civitavecchia. Moreover the 15th century Torre Niccolina, abandoned due to the withdrawn of the sea, was substituted by *Torre Alessandrina* (1662). In the South the stretches of coast between Anzio-Nettuno and Circeo and between Circeo and Terracina were connected with the construction of respectively *Torre Foce Verde* (1660) *Torre Fogliano* (1622) and *Torre Vittoria* (1624), *Torre Olevola* and *Torre Badino* (1610). During the 18th century the defense system remained unchanged, apart from the reconstruction of inadequate towers (as in the case of Torre Olevola) and the substitution of unusable towers, such as the Torre Alessandrina at the Tiber outlet which was replaced by *Torre Clementina* in 1773. During the 19th century, with the Napoleonic invasion, the defense system of the Pontifical State participated actively in the process of transformation and faced numerous assault attempts of the English navy. The organization of the network, even if radically transformed in its use and function, remained structurally unchanged till the first half of the 20th century. During the last World War a number of towers, especially in the South, were completely destroyed or irreparably damaged and the effects of these actions can still be noticed today. Nowadays most of the structures are waiting for conservation, consolidation measures or rehabilitation in order to revive the glorious events they witnessed during their lifetime.

3. The 16th century watchtower model

As mentioned in section 2, the 16th century watchtowers (fig.2) reproduce a predefined design advocated by the *'Consitutio de aedificandis turribus in oris maritimis'.* Guglielmotti [3] summarizes the characteristics of the tower as follows: square plan tower, with 10m side, 20 m high, 3-4m thick walls, inclined façades under the cordon, tall doorway above the cordon, and external staircase with a movable bridge connecting to the entrance. Three vaulted floors: one for storage, one for living and one for shooting. Internally, a winding staircase connects the different levels.

Figure 2. Examples of 16th century watchtowers. From left to right: Torre Olevola, Torre Epitaffio, Torre Vittoria and Torre Pesce.

According to a report done in 1677 it is possible to highlight further details [4]: the walls present a three-leaf section with a 40cm wide stone blocks masonry leaf in the interior part and a leaf of irregular stone masonry finished with a layer of brick masonry in the exterior part; the nucleus is made of rubble masonry with pozzolanic mortar. The shape is very distinctive: trunk-pyramidal base for distributing the weight over a larger area of soil (often with quite weak resistance on the seaside) and distance the enemies to a more vulnerable position; parallelepiped body over the base. The structures are oriented so to maximize the shooting range (usually with one corner to the seaside). Wall connections are strengthened with travertine blocks at the corners. The idealized model of the 16th century watchtower (fig.3) couples the information given by the conspicuous historical accounts (as outlined above), with data obtained from the in-situ survey of Torre Gregoriana made by the authors [5] (fig.4), so to more effectively represent the general characteristics of the real structure.

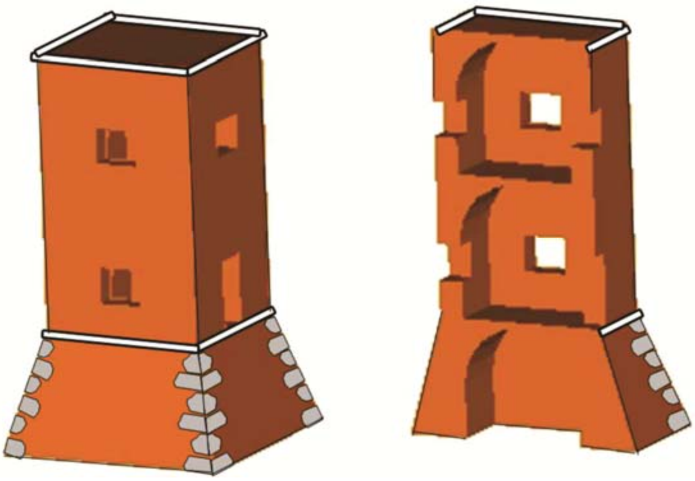

Figure 3. Idealized model.

The model presents a simplified geometry: 10x10m vaulted trunk-pyramidal base, 5m tall, with masonry walls of width varying between 2.5m at the bottom and 1m at the top of the base. Two vaulted floors above the base, each with a height of 6m per level, with 7x7m perimeter plan and 1m thick masonry walls. The openings, which are located in the floors above the base, are symmetrically distributed with the exception of the larger doorway which is found at the first floor level. The geometry of the vaults is obtained from a geometrical analysis of the surveyed structures presented in table 1. The rise/span factor oscillates between 1:3 (Torre Gregoriana) and 1:2 (model). It is possible to observe how the shape of the vaults in real structures is that of a slightly shallow arch converging towards a semi-circular one from the base to the second floor. The thickness to span ratio ranges between 1:20 (Torre Gregoriana and Torre Olevola) and 1:17 (model). For simplicity a semi-circular shape with constant vault thickness is assumed.

				Gregoriana	Olevola	Model
VAULTED FLOORS	Base	rise/span	*(-)*	0.35		0.50
		t/span	*(-)*	0.06		0.05
	1st floor	rise/span	*(-)*	0.38		0.50
		t/span	*(-)*	0.08		0.05
	2nd floor	rise/span	*(-)*	0.41	0.42	0.50
		t/span	*(-)*	0.08	0.08	0.05

Table 1. Geometry of vaults.

Figure 4. Visual inspection: wall cross section.

The material characteristics for the proposed structural model are determined in accordance with section 11.5.2.3, of the Italian building codes for seismic retrofitting [6]. The level of knowledge considered for the building is 1, as limited in-situ testing is available. The mechanical parameters are therefore set as the minimum of the range provided in table 11D.1 and corrected by applying factors in table 11D.2. The masonry typology selected presents the following characteristics: compressive strength f_m= 1.1 N/mm^2, shear strength τ_0= 0.035 N/mm^2, Young's modulus E=1020 N/mm^2, shear modulus G=1070 N/mm^2 and specific weight w= 20kN/m^3. These values do not include the application of any correction factors.

4. Structural Assessment

The structural assessment is based on a global analysis of the watchtower model. The methodology consists in carrying out a simple index assessment, a linear dynamic and a non-linear static analysis. The evaluation of simple geometric indexes provides a preliminary characterization of the vulnerability of the structure. The linear dynamic analysis helps defining the design stresses which develop when a response spectrum is applied. A nonlinear static analysis provides clues concerning the real behavior of the structure when subjected to a horizontal loading. Both analyses are conducted using a FE model developed in the software package ADINA [7]. The assessment refers to the location along the coast which presents the most hazardous scenario, applying the highest PGA found and the worst soil category.

The reference response spectrum employed is the one for ultimate limit state (as per section 3.2.5, [6]). The maximum horizontal ground acceleration is a_g= 0.2g (2A zone, [8]), the considered soil is of type D with factor S= 1.35, and the periods of separation between the different branches of the spectrum TB= 0.2; TC= 0.8 and TD= 2 (T. 3.1, [6]). The design spectrum is reduced by a behavior factor q= 2, which is the average of the typical range for unreinforced masonry (T. 9.1, [9] and 11.5.4.2 [6]). The damping is considered equal to 5 percent.

4.1 Simple indexes

In-plane and out-of-plane indexes are calculated for two real structures (Torre Gregoriana and Torre Olevola), whose geometrical characteristics are known, and for the proposed model. Such approach allows a direct comparison of the results, validating the geometrical assumptions of the model. All indexes refer only to geometrical parameters. Factors that are not taken into account are the type of construction, the quality of the walls and the connections, and the presence of lateral restraining structures. A symmetric structure, with rigid diaphragm floors and dominant in-plane shear failure of the walls, is therefore assumed [10]. The indexes include:

- *In-plane indexes*: plan area ratio $\gamma_{1,i}$, as the ratio between the area of the earthquake resistant walls in each main direction (transversal x and longitudinal y) and the total plan area of the buildings; area to weight ratio $\gamma_{2,i}$ which provides the ratio between the area of the earthquake resistant walls in each main direction (again, transversal x and longitudinal y) and the total weight of the construction; base shear ratio, $\gamma_{3,i}$ which provides the ratio between shear resistance of the structure (based on the contribution of resistant walls) and the base shear due to seismic loading (calculated as an equivalent static load).
- *Out-of-plane index*: thickness to height ratio of perimeter walls, γ_4.

			Gregoriana	Olevola	Model
IN-PLANE	$\gamma_{1,x;y}$	*(-)*	0.38	0.36	0.30
	$\gamma_{2,x;y}$	*(m^2/MN)*	2.91	2.66	2.93
	$\gamma_{3,x;y}$	*(-)*	1.68	1.39	1.34
OUT-OF-PLANE	$\gamma_{4,x;y}$ (base)	*(-)*	0.54	0.41	0.35
	$\gamma_{4,x;y}$ (body)	*(-)*	0.46	0.42	0.25

Table 2. Simple indexes analysis.

The non-dimensional index $\gamma_{1,i}$ is the simplest one, being associated with the base shear strength. Values obtained in table 2 range from 38 to 30% for both x and y directions, being the structure symmetrical. The towers satisfies the threshold for $\gamma_{1,i}$ set to values up to 5-6% for regular structures with rigid floor diaphragms [9], or to 10% in case of high seismicity [11]. Index $\gamma_{2,i}$ is associated with the horizontal cross-section of the building, per unit weight. The height of the building is taken into account, with the disadvantage that the index is not non-dimensional and hence must be processed for fixed units. The model scores the highest value of 2.93 m^2/MN (for a total volume of the structure of 564 m^3), followed by Torre Gregoriana with 2.91 m^2/MN (volume equals to 778 m^3) and Torre Olevola with and index of 2.66 m^2/MN (864m^3). The values obtained are identical in both x and y directions, being the area of earthquake resistant walls the same. Higher ratio indicates safer structure, or in other words larger relation resistant walls/weight and lower heights. The results comply with the minimum value of 2.5 m^2/MN, adopted for high seismicity zones [11]. Index $\gamma_{3,i}$ relates the shear resistance of the towers and the base shear produced by an equivalent static load for the seismic effects. The indexes $\gamma_{3,i}$ shown in table 2 include 1.68 for Torre Gregoriana, 1.39 for Torre Olevola and 1.34 for the proposed model. These values are again similar for both loading directions. The minimum value of $\gamma_{3,i}$ equal to 1 is satisfied [11].

Index $\gamma_{4,i}$ refers to the ratio between the thickness and the height of perimeter walls. Being these dimensions quite different for this typology of structure, two separate indexes are calculated for the base and body walls. The index for the base walls is higher than that for the body. The former one ranges between 0.54-0.35 while the latter spans between 0.46 and 0.25. The calculated indexes satisfy the maximum allowable h/t= 10 [6] Furthermore also the geometrical requirements $t_{min} > 300$mm, and $l/h > 0.5$ set for earthquake resistant walls in table 8.1 [6] are satisfied.

Indexes $\gamma_{1,i}$, $\gamma_{2,i}$ and $\gamma_{4,i}$ are independent on PGA (for fixed safety level therefore there would be increasing indexes values with increasing seismic action) while index $\gamma_{3,i}$ takes into account the seismic demand (the index value is constant in different seismic zones). It should be noted that all indexes threshold values, set for a PGA 0.25g, are more conservative than those which apply for the ground acceleration at the location under analysis (0.2g). The in plane indexes calculated for the model fall within 10-20% of those found for the real structures. This allows assuming a similar behavior of the model with respect to the real structure. The out of plane index for the model is about 40% lower than that for Torre Gregoriana and Torre Olevola, ensuring the analysis of a more critical context in the out of plane behavior.

4.2 Linear dynamic analysis

The modal analysis allows defining the maximum structural response induced by the application of the design response spectrum, as defined in the introductory paragraph of section 4. The modes considered are those necessary for reaching a minimum of 85% of the total participating mass, with a participating mass for each mode larger than 5% (fig.5). The most relevant modes include: mode1 (1.2Hz), mode 4 (3.4Hz) and mode 10 (7.6Hz) in the x direction, with a participating mass of respectively 45.3%, 17.5% and 15.9%; mode 2 (1.2Hz), mode 5 (3.9Hz) and mode 9 (7.5Hz) in the y direction with related participating mass 45%, 18.7% and 14.4%. For low frequencies the modal shape presents a rigid translation of the body of the tower with respect to the base, intermediate frequencies induce one-directional bending (one-point inflection) and higher frequencies produce an S modal shape (two-point inflection).

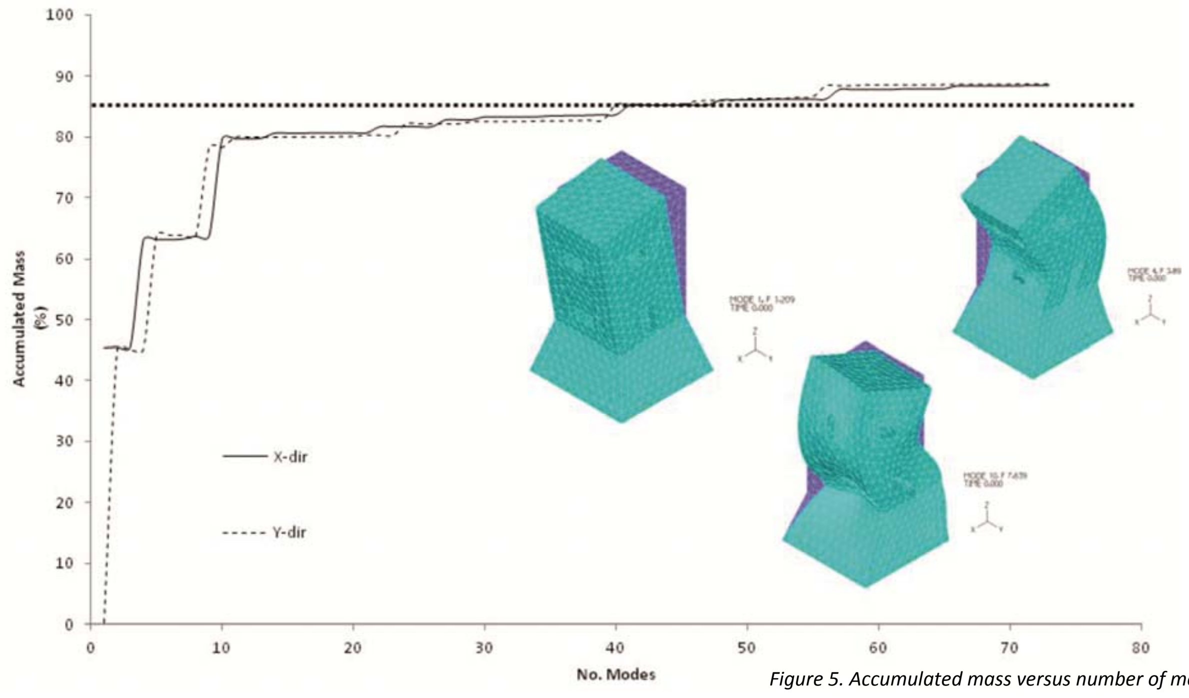

Figure 5. Accumulated mass versus number of modes.

The superimposition of the effects of the different modes is calculated by a complete quadratic combination (CQC) as not all modes differ by 10% at the total participating mass. The application of the spectrum is along the X axis, as the area of the openings is greater with respect to the one in the Y direction. The response displacements due to the application of the response spectrum are visualized in figure 6: these are respectively 0.008m, 0.0002m and 0.002m for x, y and z directions. In order to approximate the real displacement corresponding to the ultimate limit state for the watchtower it is possible to multiply the results of the modal analysis by the factor q and the relevance factor of the structure (γ=1, T4.3 [6]), (section 4.8 [6]). In the case of x direction, the corrected value is about 0.016m.

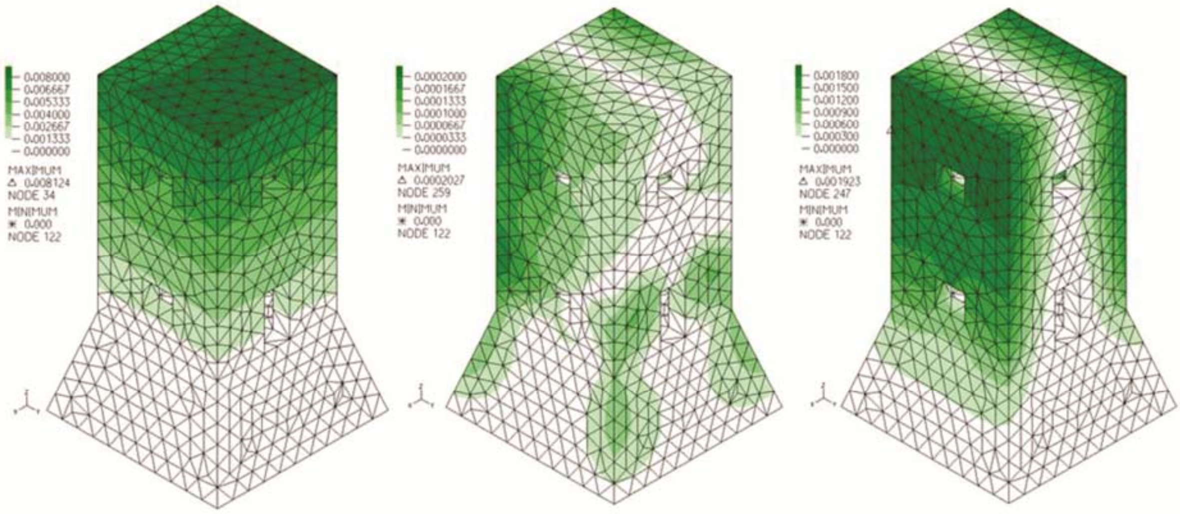

Figure 6. Displacement. From left: x-displacement, y-displacement and z-displacement.

Resulting stresses and strains are presented in figures 7 and 8. Large vertical stress develop at the contact area between the body and the base on the out-of-plane walls, while shear stress concentrates on the corners of windows and the door opening on the shear walls.

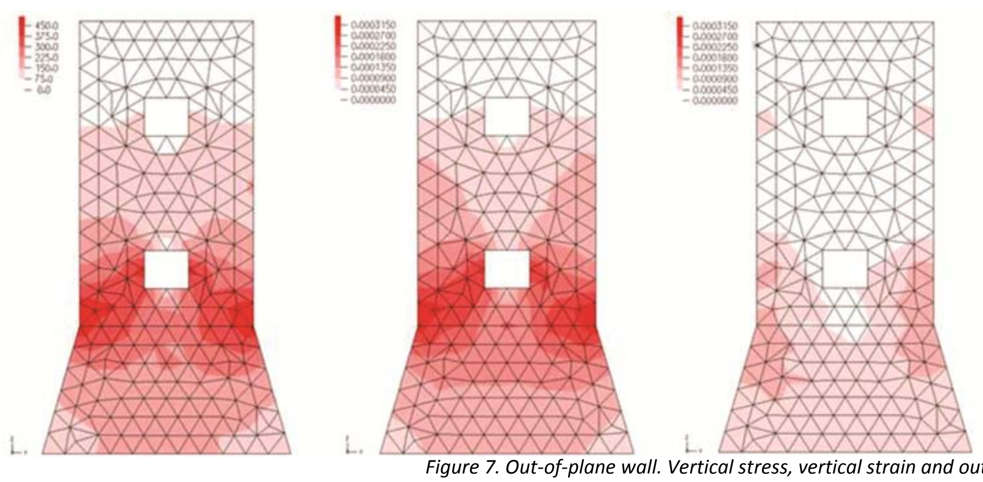

Figure 7. Out-of-plane wall. Vertical stress, vertical strain and out-of-plane strain.

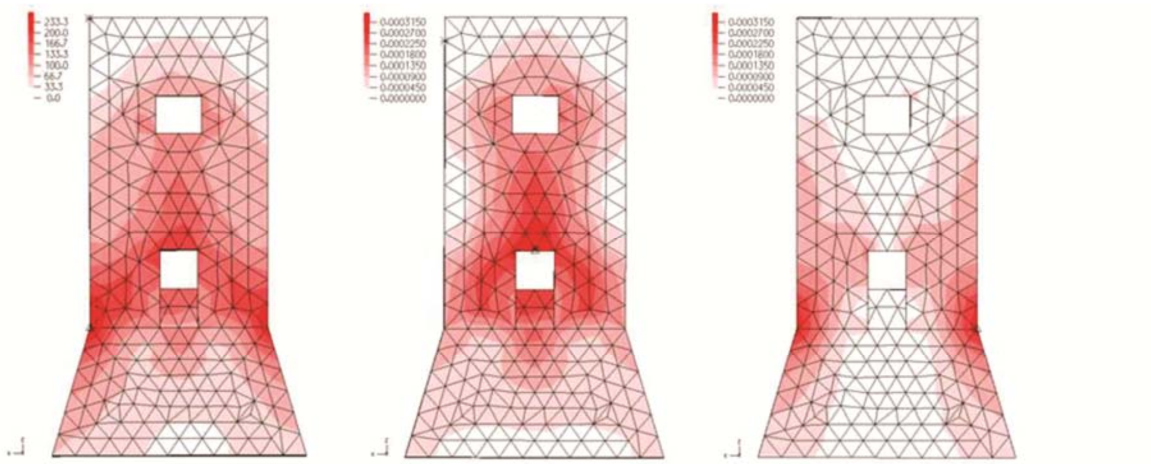

Figure 8. In-plane wall. Shear stress, shear strain and vertical strain.

The analysis shows that low frequency mode 1 (i.e. with largest period T=0.83s) is dominant. The peak of the stress along the vertical axis is found for 450 kPa (tension), even if this depends to some extent on the mesh refinement. Highest shear stress found is about 230 kPa. The results indicate that, although the global safety is not endangered, damage will develop. Most vulnerable parts of the structure are the shear walls at first floor level and the tensioning side of the tower just above base level. It should be underlined that in practice the maximum values of the different modes occur at different times and the CQC estimate constitutes only an approximation of the maximum forces the structure might be subjected to at some specific point in time of the oscillatory motion.

4.3 Nonlinear static analysis

Nonlinear static analysis allows determining the failure load as well as the ductility capacity of the structure. Such analysis is carried out by applying a monotonic mass proportional loading in the x-direction (critical direction). The displacement of the control node d_c is monitored and then plotted against the base shear resistance F_b. The resulting F_b - d_c curve is shown in figure 9. The limit conditions for the linear behavior, individuated at point A, occur for a horizontal force multiplier of α= 0.41 with the formation of plastic hinges at the corners of the openings at the first floor. The maximum resistance of the watchtower is found at point C (F_{bu}= 6440kN, d_{cu}= 0.024m) for a horizontal force multiplier α= 0.57. At this point the body of the tower, assuming a monolithic behavior, begins overturning about the compressed top of the trunk-pyramidal base. A global shear failure mechanism occurs by diagonal cracking through openings at first floor level. The horizontal force multiplier for the activation of such mechanism, calculated by a simple kinematic model as α= 0.43 (fig.9), reasonably approximates the value found on the pushover curve. The stress along the vertical axis, the horizontal strain in shear walls and crack patterns for point C are shown in figure 10. The damage limit state (4.11.2, [6]), after which the structure is considered unusable, is found accordingly to the Italian building codes, for a differential displacement between two consecutive floor d_r < 0.016m which occurs at a multiplier of the horizontal force α= 0.52 (point B). The collapse point is assumed to be reached at 80% of the maximum base shear.

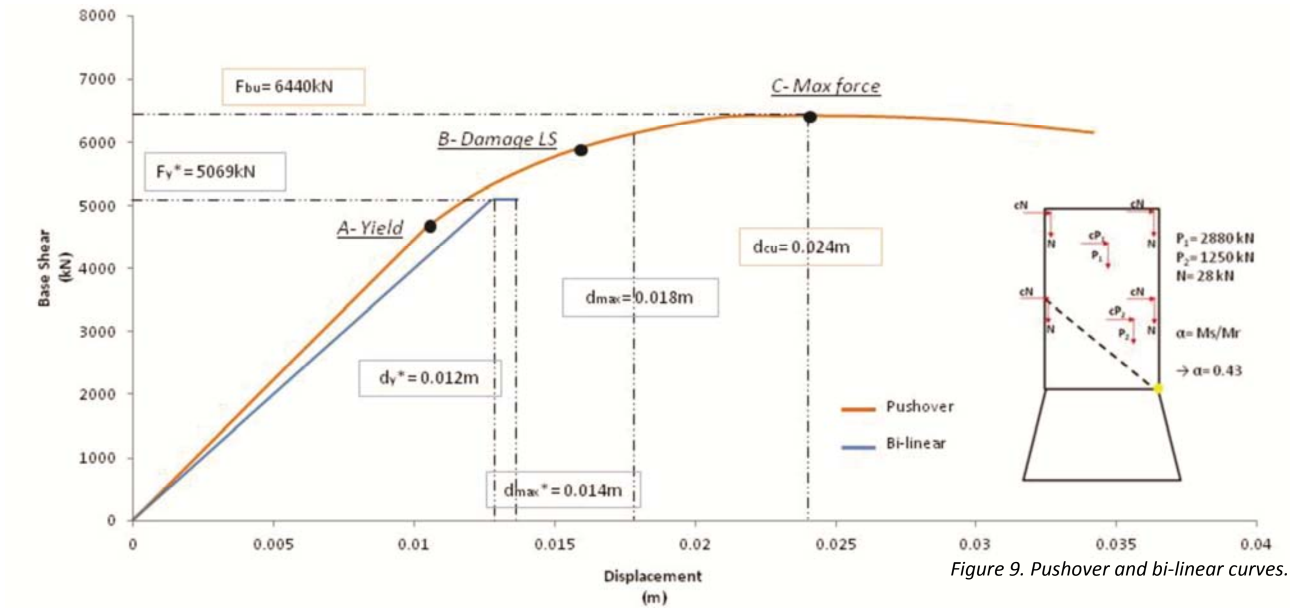

Figure 9. Pushover and bi-linear curves.

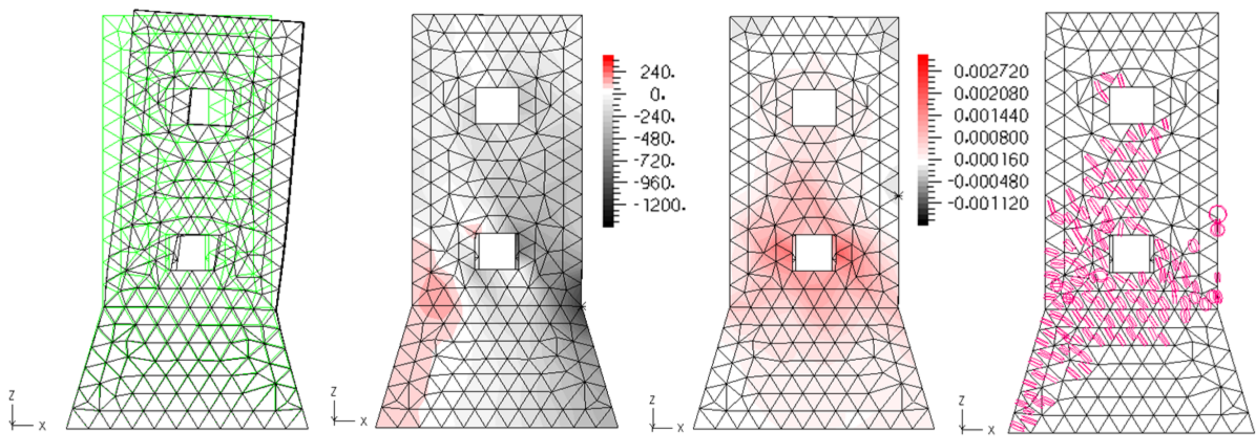

Figure 10. Deformed shape, vertical stress, horizontal strain and crack pattern.

In order to assess that the displacement capacity of the structure is larger than the displacement demand posed by the design seismic action, the pushover curve is transformed into a bi-linear force-displacement relationship for an equivalent SDOF system (F_b*- d*). After this step the maximum response of such system is determined (d_{max}*) by applying the response spectrum as described in the introductory section. The final step includes the transformation of the SDOF response into the corresponding maximum displacement of the original structure (d_{max}) whose compatibility can be verified [12]. The F_b*- d* relationship is obtained by employing the following relationships:

$$F^* = \frac{F_b}{\Gamma} \quad ; \quad d^* = \frac{d_c}{\Gamma} \quad ; \quad \Gamma = \frac{\sum m_i \Phi_i}{\sum m_i \Phi_i^2} \qquad (1);\ (2);\ (3)$$

The bi-linear curve, shown in figure 9, is plotted by determining the position of its yielding point. Considering equation (3), where Φ_i is the normalized vector of the first mode of vibration of the structure and m_i the mass matrix, the participation factor is equal to Γ= 1.31. From equation (1) the bi-linear yield force, for a maximum force F_b= 6440kN, is calculated as F_y*= 5069kN and the corresponding displacement, determined by using an equal-energy criterion, is d_y*=0.0128m. The multiplier of the horizontal force at this yielding point is α= 0.45. The elastic period is obtained by the relation:

$$T^* = 2\pi\sqrt{\frac{m^*}{k^*}} \qquad (4)$$

where k* is equal to the secant stiffness of the equivalent system (F_y*/d_y*) and m*= Σ$m_i\Phi_i$= 620t represents its modal mass. For the period T*= 0.25s calculated from equation (4), the displacement of the system is given by:

$$d_{\max}^* = S_e(T^*)\left(\frac{T^*}{2\pi}\right)^2 \qquad (5)$$

verifying that, being T* < T_c= 0.8, the ratio q* between the force of the elastic response of the equivalent system and its yielding force is less than 1:

$$q^* = \frac{S_e(T^*)m^*}{F_y^*} \leq 1 \qquad (6)$$

For S_e(T*)= 0.338g, determined by the design spectrum parameters in the introduction of section 4, equation (6) is satisfied being the ratio q*= 0.04. The maximum displacement of the equivalent system given by equation (5) is d_{max}*= 0.014m. This value is transformed into the displacement demand for the structure using equation (2) d_{max}= d_{max}* Γ. The so computed value of the displacement d_{max}= 0.018m can be compared with the ductility capacity of the structure determined by the F_b-d_c curve. It should underlined that d_{max} falls within a reasonable range from the value of the maximum x displacement determined in the modal analysis (d_{max}=0.016m). The performance of the structure can therefore be considered sufficient at a global level, taking into consideration however the limitations of the model due to the approximation of geometrical and material characteristics.

5. Conclusions

According to the simple index analysis the structure representative of the 16th century towers in the defense system of the Pontifical State in Italy is very stocky due to its massive walls. All calculated indexes satisfy the set of thresholds, predicting a sufficient performance of the structure in case of earthquakes. The similarity in the results of geometrical ratios allows also concluding about the good approximation of the proposed model. The linear dynamic analysis proves a safe global behavior of the structure when subjected to the design response spectrum. However high stress concentrates at specific points in the structure (especially around the openings in walls), suggesting the possibility of damage. The nonlinear analysis confirms a shear failure mechanism with a global overturning of the body around the base. In both analyses the application of the reference response spectrum induces exceeding the damage limit state but still below the activation of the failure mechanism.

The proposed model, based on historical records and in-situ investigation, provides a good approximation of the 16th century watchtower. However some limitations might derive from the scarce knowledge concerning the mechanical parameters and damage level of the structures. Deeper knowledge would be therefore beneficial in updating the model and assessing the real capacity of the structure. In conclusion, the 16th century watchtower is safe against a global failure due the most hazardous scenario at the site and for the most critical geometrical and material characteristics of the idealized model. However, local failure mechanisms (e.g. wall leaf detachment, corner overturning and vaults opening) should be considered in more detail and further analyses carried out to verify the need for local interventions.

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